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DESIGN OF LOW VOLUME ROADS

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DESIGN OF LOW VOLUME ROADS

TO: J. F. McLaughlin, Director
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FROM: H. L. Michael, Associate Director
Joint Highway Research Project

The attached Technical Paper "Design of Low Volume Roads" has been prepared by Professor E. J. Yoder and Graduate Assistant A. A. Gadallah for presentation at a conference on low cost roads in Kuwait. The paper results from no particular research of the University but from the accumulated experience, research and study of the authors.

The paper is presented to the Advisory Board as information because of its pertinence to highway problems in Indiana and elsewhere in the world. It should also be of interest to the county highway officials of Indiana and will also be distributed through the Highway Extension and Research Project for Indiana Counties.

Respectfully submitted,

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SYNOPSIS

The literature abounds with reports dealing with all phases of road engineering. It is true that major effort has gone into study of roads and airports that carry high traffic volumes. Nevertheless, it is a known fact that there is a large mileage of roads on a world wide basis which carry far less than expressway volumes of traffic.

This paper summarizes some concepts that have been developed relating to design of low volume roads. A great deal of attention has been directed in recent years towards the Systems Analysis approach to road design. This paper presents a discussion of the sensitivity of traffic, materials, and stage construction.

The basic philosophy behind the paper is that low cost roads should be designed on an "areal basis". This is true inasmuch as utilization of local materials and the establishment of design units for an area form the

backbone of the decision-making process of design. Particular reference is made to use of stage construction and determination of subgrade properties based upon climatic and other environmental considerations of the area under consideration.

It is brought out in the paper that some of the factors used in the analysis are relatively insensitive and therefore rough estimates of their effects are all that is needed in the planning process. Other factors, however, become relatively significant and must be determined with greater accuracy . Methods of estimating some of the parameters for design areas (or geographical units) are presented in the paper.

The basic area design unit proposed by the authors is based on a number of factors including geology and soil type of the area, materials available for construction, climate, estimates of amount of traffic within a geographic area and others. In many respects, the delineation of a design unit is qualitative in that it requires a thorough understanding of many factors that are difficult to evaluate within the area. Similar areas require similar design parameters.

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INTRODUCTION

During the past ten to thirty years concepts dealing with pavement design have taken dramatic steps forward. In the early stages of design, the engineer was primarily concerned with an evaluation of the subgrade and of the traffic that might use the road. Design methods were primarily empirical wherein correlations were established between traffic and thickness of pavement for various subgrade conditions.

In recent years, a great deal of attention has been focused on the design of high traffic roads throughout the world. These designs have in part been empirical, but on the other hand, recent research has considered theory of stress distribution, and in particular, fatigue characteristics of asphalt surfaces. As a natural consequence, the amount of traffic which is to use the road has been given primary consideration.

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However, it is known that on a world-wide basis the number of miles of low traffic roads far exceeds the miles of high traffic roads. Approximately two-thirds of the rural roads in the United States today carry less than 400 vehicles per day.

A basic need has existed for a period of years to distinguish between concepts which differentiate between design of "low volume" and "high volume" roads. Recent emphasis has been placed on the "systems approach" in which an economic analysis is made of road construction and the cost of the road construction has been optimized relative to least cost. This optimization process has been called systems analysis and in many cases, it involves a very extensive series of computations in which a wide variety of alternates are considered. For low volume roads these extensive computations are often unnecessary due to the insensitivity of some of the factors.

It is a primary purpose of this paper to summarize some of the basic principles involved in road design and construction with particular emphasis on roads that carry relatively low volumes of traffic. Reference will be made in the paper to recent research which has evaluated the systems approach on a general basis so that criteria can be established to fit a variety of conditions.

Especially for low volume roads it is not always necessary to make a detailed economic analysis of every road. Rather, sufficient data have been accumulated over the past decade to permit the engineer to rely on criteria which enables him to make an analysis for a given situation without resorting to detailed analysis and justification. In the design of a specific highway, the analysis can involve an extensive evaluation of a large number of inconsequential factors unless the engineer is prepared to rely upon past experiences and to group his design factors into "design units".

A basic theme of this paper is that one approach is to design a road on areal basis taking into account the environment of the area, soil conditions and traffic. On this basis an economical design can be derived to fit a given condition.

THE DESIGN PROCESS

Figure 1 shows in flow diagram form the design process. On the left hand side of the figure, a series of input variables are listed which include fundamental stress strain analysis of the paving components, load and traffic analysis, environmental factors and evaluation of material properties including soil and base stabilization. The next step in the design process involves decisions in which the variability of pavement properties and of the area are considered. From this a specific design value is selected. The third step is the selection of the design.

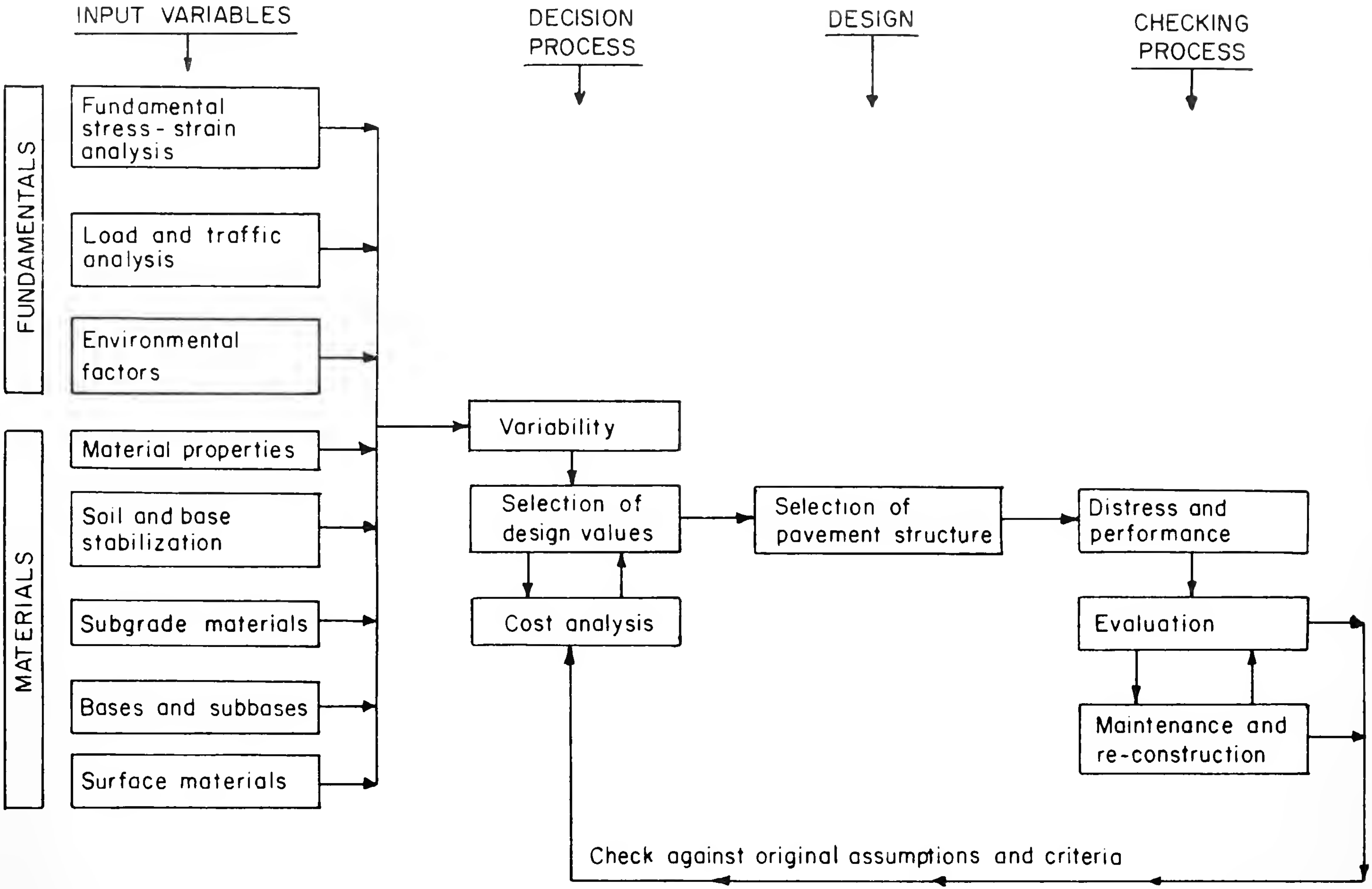


FIG. I PRINCIPLES INVOLVED IN THE DESIGN PROCESS

The last and, perhaps the most important phase of the process, is that of checking the design against the assumptions that were made in the previous phases. This checking process includes distress and performance surveys, evaluation of the existing facility, maintenance and reconstruction. This checking process then must be fed back through the original design and the various alternates compared.

To facilitate his decisions, it is necessary for the engineer to estimate the performance and evaluation of the pavement so that this can be fed back into the decision making process at the time that the design is made. This bypassing of the checking process requires that the engineer rely heavily upon past experiences as well as upon certain assumptions he must make regarding the probable behavior of the pavement.

As a part of the decision making process a cost analysis is generally made and adjustments in the design are made depending upon the final cost. The design is relatively insensitive to several factors (especially for low volume roads) and, therefore, it is possible for the engineer to estimate these with wide variation without throwing too much error into the analysis. Hence, the relative sensitivity of the analysis itself will be discussed herein with the hope of putting these factors into perspective.

THE ECONOMIC ANALYSIS

In recent years a great deal of attention has been focused on the economic analysis itself. This is particularly true of low volume roads. This is doubtless the correct approach since it forms the basis for sound engineering judgement and final decisions. However, the economic analysis, as any analysis, is only as accurate as the data input.

The sensitivity of the various economic factors in the analysis becomes extremely important in the engineers decision-making process. For a comprehensive evaluation of the economic analysis for highways the reader is referred to the works of Winfrey (Reference 31). Likewise, the World Bank (11,15) has been a leader in developing economic analysis of road construction. A great deal of research has been conducted by the Road Research Laboratory in England (16,24).

For purposes of this paper, the analysis is concerned solely with the design of the physical structure itself; no reference will be made per se to the economic consequences of constructing a given facility in an area.

<u>Interest Rate and Its Effects</u>. As in any economic analysis, rates of interest available for money to be spent at some later date greatly influence the total cost of the investment. Interest has the effect of reducing maintenance costs, and giving an advantage to deferring

costs to a later date. Use will be made herein of several terms commonly used in the standard analysis.

Present Worth. If P dollars are invested now, this money will have additional worth at the end of the investment period as shown in Equation 1.

$$M = P\ (1 + i)^n \tag{1}$$

$$P = \frac{M}{(1 + i)^n} \tag{2}$$

$$P = M\ (PW) \tag{3}$$

where:

$$PW = \frac{1}{(1 + i)^n} \tag{4}$$

i = Interest rate

n = Number of years

Average Annual Cost. In the economic analysis, costs are sometimes (not always) expressed as an average annual cost even though the money is spent in a lump sum. The average annual cost is given by Equation 5.

$$AAC = I_c(CR)_n \tag{5}$$

where:

CR = Capital Recovery Factor

$$CR = \frac{i(1 + i)^n}{(1 + i)^n - 1} \tag{6}$$

I_c = Initial cost

Salvage Value. A road that is designed to last for a period of n years may or may not have some salvage value at the end of the analysis period depending on several factors. In some cases, the surface of the road itself will have some value, but changes in right-of-way, deterioration of structures, culverts and other features leave this point open to question. If the salvage value is designated by S, the total present worth is:

$$\text{Total Present Worth} = I_c - S(PW_n) \qquad (7)$$

where (PW_n) is the Present Worth Factor at the end of the analysis period.

Major Maintenance Costs. Major maintenance costs are paid out in lump sums at specified periods during the life of the pavement. In addition, however, the road should receive routine maintenance throughout the life of the pavement. If the major maintenance at the end of n_1 years is designated M_m and the total analysis period is n years, the present worth of the costs are:

$$\text{Present Worth} = I_c + M_m(PW)_1 - S(PW)_n \qquad (8)$$

Values of routine maintenance costs and/or road user costs can be substituted in the equation in lieu of M_m. Hence, the general form becomes:

$$\text{Total Present Worth} = I_c + \sum_1^n (x)(PW) - S(PW)_n \qquad (9)$$

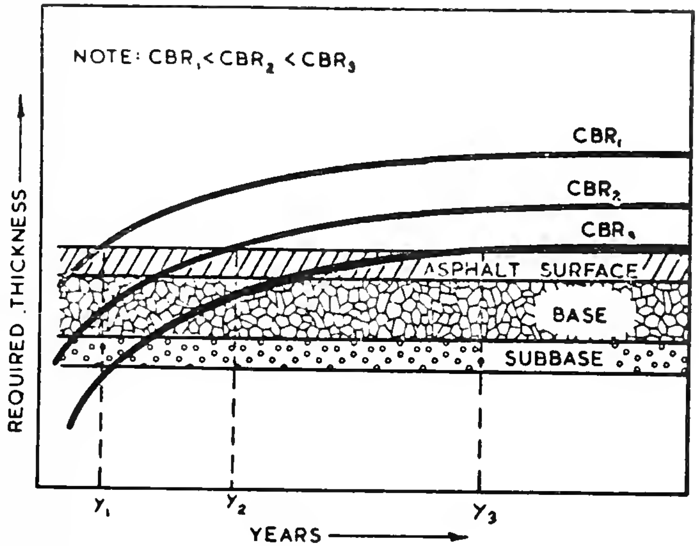
where x becomes values of costs applied at intervals regardless of their type and PW is the present worth factor for the years at which "x" is spent.

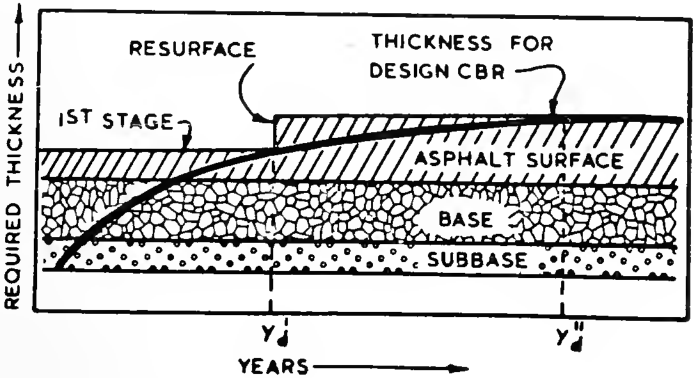
ROUTINE VS. MAJOR MAINTENANCE COSTS

At the outset it is necessary to distinguish between "routine" and "major" maintenance costs. In the first case, routine costs are the day-to-day expenditures that the government must place on a road to keep it serviceable. On the other hand, major costs are planned costs which are paid out on a lump sum basis at specific periods of time. Figure 2 illustrates this concept in diagramatic form.

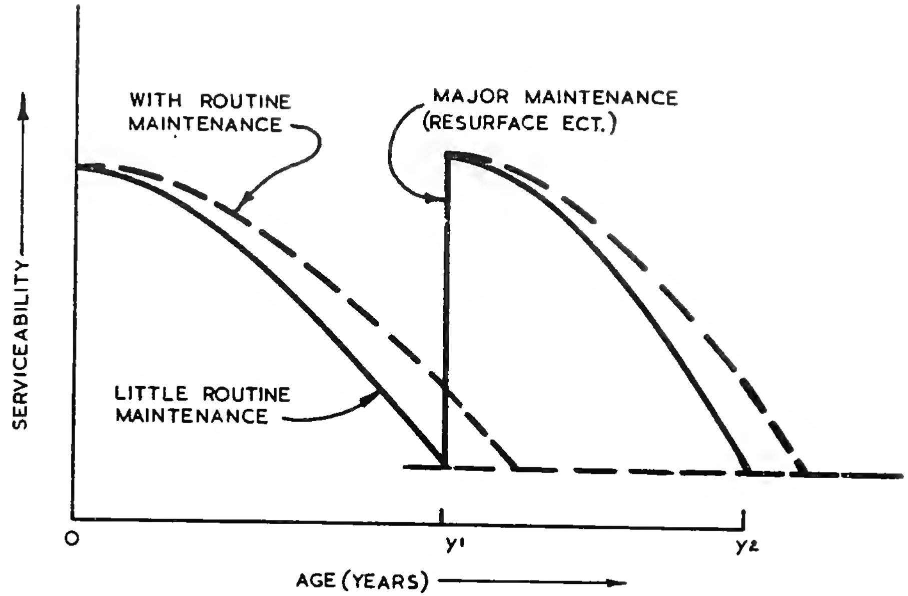
Reference is made to the serviceability concepts that were developed on the AASHO Road Test (13). The serviceability concept, in general, refers to the ability of the road to perform its intended function at any instant of time. As noted in Figure 2, when the road is first constructed it will have high serviceability but, as time progresses, this serviceability decreases. At some specified interval of time the road may be resurfaced or replaced and the serviceability brought back to its initial value.

The rate of deterioration of the road is dependent upon adequacy of the routine day-to-day maintenance. As shown in Figure 2, if the routine maintenance is used, the road will reach a terminal serviceability at a later period of time than if no routine maintenance is put into the road.





GENERAL RELATIONSHIP BETWEEN AGE AND REQUIRED THICKNESS



GENERALIZED SERVICEABILITY VS. AGE

FIG. 2 MAJOR AND ROUTINE MAINTENANCE

Perhaps one of the most tenuous factors that the engineer must estimate is the cost of routine day-to-day maintenance. Exact estimations of this cost factor are perhaps not too important. However, it is important that routine maintenance be put into the road. The point is made that estimations of routine maintenance costs are very difficult to make and therefore the engineer must approach this particular factor with some caution. Since many times no routine maintenance is put into the road at all; this factor, then, can nullify the economic analysis.

On the other hand, it is generally easy to estimate major maintenance costs since this can be done through the use of conventional design methods. Therefore in an analysis of the various factors which influence the economic evaluation distinction must be made between the nebulus nature of estimating routine costs as opposed to the relatively certain techniques for estimating major costs. It is essential that the reader differentiate distinctly between these two types of costs.

ROAD USER COSTS

For the remainder of this paper use will be made of information furnished by the paper by DeWeille (11) for estimation of running road user costs. Road user costs proposed by DeWeille are differentiated upon the basis of paved and non-paved roads for various types of vehicles.

<u>Added Road User Costs</u>. In addition to running road user costs there is an added cost that might accrue due to the fact that the facility is shut down for a period of time for maintenance. This factor becomes particularly important for high traffic roads since many times the shutdown of the facility itself may be so costly to preclude any maintenance procedures at all. On the other hand, for low volume roads (less than 100 vehicles per day) the matter of added user costs become minimal and they have been disregarded in the analysis to follow.

SENSITIVITY OF THE COST FACTORS

The economic analysis presented in previous paragraphs produces a numerical answer which can be used as a guide by the engineer in establishing his design. These must be used as <u>guides</u> only, since the factors are subject to judgement of the analyst and are interrelative with many other factors.

Pavement design in its strictest sense has historically been concerned with evaluation of soil strength and estimation of traffic to be applied to the pavement; from these data the engineer selects a structure. Generally, the design life of the pavement structure is assumed, or at least implied in the analysis. If one considers the matter of serviceability, and in particular the factor of personal opinion as it influences serviceability, it becomes apparent that a specific design can vary as much as 100 percent and that little argument can be propounded to

substantiate or negate the hypothesis upon which the design is predicated.

When considering alternate designs some of the factors become critical whereas others become insensitive; the designer must keep these in mind at all times. The following paragraphs present a short discussion of the factors which influence the economic analysis.

<u>Effect of Salvage Value</u>. The salvage value of the pavement investment is, as shown in Equations 7 through 9, reduced to the present worth of the salvage value at the end of the analysis period and it decreases as the analysis period increases. If, for example it is assumed that the salvage value is 30% of the initial cost at the end of 20 years, and assuming an interest rate of 10%, the present worth factor (PW) at 20 years is 0.1486; for a 40 year period it is 0.0221. This means that for the assumed conditions the present worth of the salvage is as shown below:

At 20 years:

$$\text{Present worth of salvage} = I_c(.30)(0.1486)$$

$$= 0.0445 I_c$$

The effective initial cost, expressed as a percentage of the actual value I_c, is equal to

$$100 - 4.45 = 95.55\%$$

At 40 years:

Present worth of salvage = $I_c(.30)(0.0221)$

$= 0.00663I_c$

The effective initial cost, expressed as a percentage of the actual value I_c, for this case is equal to

$100 - 0.66 = 99.34\%$

If the analysis period is taken to be as long as 40 years, the salvage value can be assumed to be equal to zero. Even for lesser analysis periods, salvage value many times has little effect on the final decision. For short periods it can become more significant.

Effect of Added User Cost. Table 1(a) shows the effect of several variables on optimum staging, wherein the added user costs brought about by shut-down of the facility is of concern. These data are from Yoder, et. al. (33). For low traffic roads, especially for good subgrades wherein relatively thin sections can be used, the added user costs resulting from maintenance is of practically no consequence and has little effect on the analysis.

For very high traffic roads, the added user cost becomes significant and many times it is the over-riding factor which dictates the design.

(a) Effect of Added User Costs (Interest Rate is 13% and Traffic Growth is 6%)

Subgrade CBR	Initial Yearly EAL*	Added Costs Considered?	Optimum Stage
(%)			(years)
2	5×10^4	Yes	9
		No	9
	5×10^5	Yes	20
		No	15
11	5×10^4	Yes	10
		No	10
	5×10^5	Yes	17
		No	14

(b) Effect of Interest Rate (Added User Costs are Considered and Rate of Traffic Growth is 6%)

Subgrade CBR	Initial Yearly EAL*	Interest Rate	Optimum Stage
(%)		(%)	(years)
2	5×10^3	6	10
		13	10
		20	8
	5×10^5	6	35
		13	20
		20	12
11	5×10^3	6	10
		13	9
		20	7
	5×10^5	6	35
		13	18
		20	12

(c) Effect of Traffic on Costs (Added User Costs are Considered and the Interest Rate is 6%)

Subgrade CBR	Initial Yearly EAL*	Traffic Growth	Optimum Stage
(%)		(%)	(years)
2	5×10^3	(all)	10
	5×10^4	2	10
		6	12
		10	13
	1×10^5	2	12
		6	18
		10	25
11	5×10^3	(all)	10
	5×10^4	2	10
		6	10
		10	10
	1×10^5	2	10
		6	11
		10	14

*Equivalent 18,000 pound single axle loads

TABLE I SENSITIVITY OF COST ELEMENTS

Sensitivity of Interest Rates. One of the major factors which can affect the analysis is the rate of interest assigned to the investment. This is illustrated in Table 1(b) for three rates of interest.

Increasing the rate of interest, in effect, gives a decided advantage to deferring payments on the investment for as long as possible. For interest rates as high as 20%, even for very high traffic, it can be demonstrated that stage construction is the most economical approach since this defers much of the payment of the investment to some later date. When analyzing these problems, it is best to at least make solutions for two interest rates before final decisions are made on the design to be adopted.

Length of Analysis Period. The length of analysis period to use depends, in part at least, upon salvage values that are assumed. Most economists agree that, for the sake of conservatism, the analysis period should be low.

Use of relatively low periods is further justified for pavement analysis, since it becomes necessary to make certain assumptions relative to the condition of the pavement at various time intervals (Figure 1).

<u>Sensitivity of Traffic</u>. The amount of traffic that will be applied to a road has a significant effect on the cost analysis in many cases. Table 1(c) is seen to be influenced significantly by traffic. On the other hand, for good subgrades (CBR = 11%) the effect of traffic growth becomes less sensitive.

In general, for low traffic and for relatively thin pavements, optimum staging is nearly independent of traffic, for this case being from 10 to 12 years. On the other hand, for poor subgrades which require thick pavements, the optimum staging increases as traffic rate of growth increases.

OPTIMUM STAGE CONSTRUCTION

Data in Table 1 is based upon the optimum (least cost) staging that should be considered for roads carrying various volumes of traffic. It is to be seen that for low volume roads the optimum staging is generally equal to or less than 10 years.

Further analysis not presented in Table 1 has shown for low volume roads, the traffic at which optimum staging becomes greater than 10 years is dependent upon subgrade strength, interest rate and length of analysis. In general, however, it can be stated that the optimum staging is 10 years or less. Hence, as a general rule, for low volume roads use should always be made of stage construction and the length of staging should be less than 10 years.

RELATIVE EFFECTS OF SUBGRADE STRENGTH AND ACCUMULATED TRAFFIC

In evaluating the relative effects of subgrade strength (in this case CBR) as well as that of traffic several different methods of flexible pavement design can be used. For this paper use is made of the design procedures developed by Turnbull et. al. (28). This method is the basic design method of the Corps of Engineers in the United States and has been adopted by the National Crushed Stone Association (20) for design of low volume rural roads.

The design method developed by Turnbull et. al. takes the general form shown in Equation 10

$$D = C \log EAL \tag{10}$$

where

D = Thickness (inches)

EAL = Total Equivalent Axle Loads (18,000# single)

C = Coefficient

Equation 10 was expanded (33) using regression techniques to that shown in Equation 11

$$D = (7.88 - 8.072 \log CBR + 2.203 \log^2 CBR) \log EAL \tag{11}$$

The equivalent axle loads (EAL) can be estimated using one of several techniques and it is the accumulated equivalent 18,000# single axle loads using each design lane. The summation of equivalent axle loads over the design life

of the pavement can be given as shown in Equations 12 and 13

$$\Sigma EAL = \frac{EAL_o(365)}{\log_e (1+i)} \left[(1+i)^n - 1\right] \qquad (12)$$

$$EAL_o = (F)(ADT) \qquad (13)$$

where

EAL_o = Initial daily equivalent axle loads on day road is opened to traffic.

i = Rate of traffic growth (percent per year)

F = Factor determined for an area

ADT = Average daily traffic in two directions.

For the analysis to follow various values of CBR and EAL were assumed and the sensitivity of these factors in determining the required depth of pavement were evaluated. Sensitivity as used herein is defined as the effect of varying one parameter while keeping the other parameter constant.

The values of CBR used in the analysis were 2, 6, 10, 20, 30 and 50 percent. The CBR variations considered were $\pm$ 10, 30 and 50 percent. The EAL values assumed in the analysis were 10, 10^2, 10^3, 10^4, 10^5 and 10^6 while the variations considered in the analysis were $\pm$ 10, 20, 40 and 60 percent. A computer program was utilized in analyzing each of the parameters in terms of the required pavement thickness D as given in Equation 11. The resultant change in required thickness ΔD and the percent

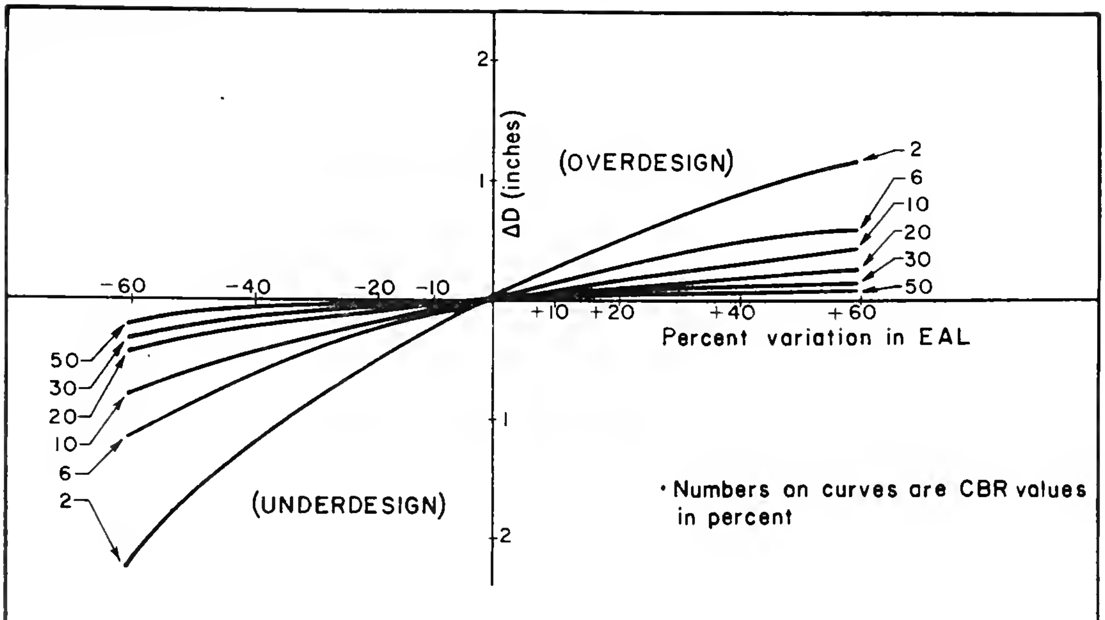
change in thickness were determined for various values of the parameters specified above.

Generally, sensitivity is expressed as a percent change with a change in any specified parameter. However, as will be explained in subsequent paragraphs this can lead to some misinterpretation since for small values of thickness, the data are distorted and, hence, primary use is made in this discussion of the finite thickness change that is caused by various changes in these parameters.

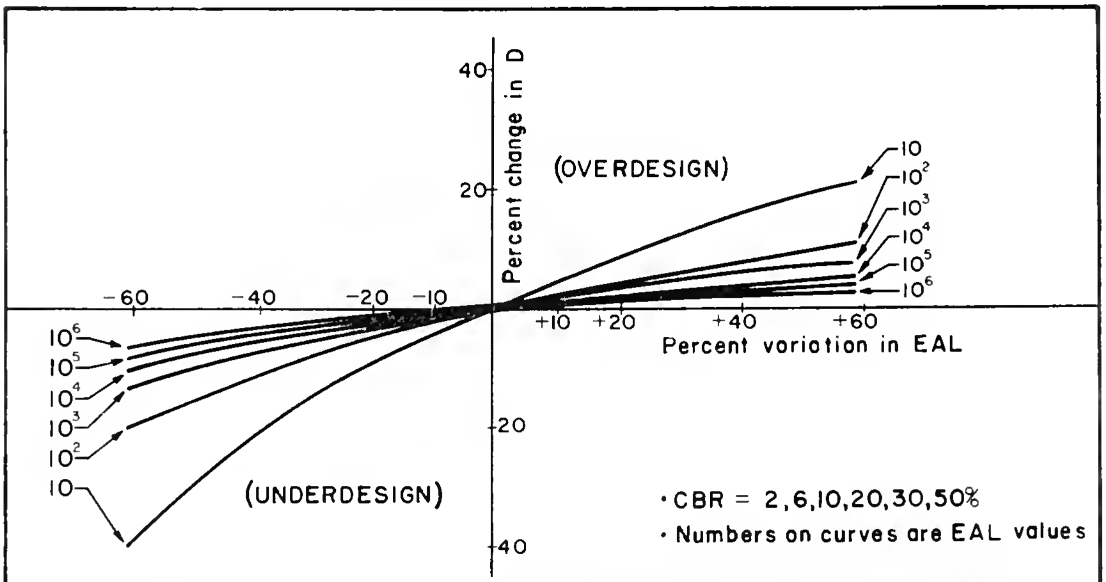
The analysis was accomplished by keeping one of the parameters given above constant and varying the other parameter as specified in the problem. Figures 3 and 4 show the resultant changes in required thickness produced by variations in each of the parameters.

Effect of EAL Variations. From Figure 3(a) it can be seen that the resulting change in thickness, ΔD, due to variations in EAL is dependent on CBR values and independent of the magnitude of EAL itself. For a given percent variation in EAL the change, ΔD, increased with a decrease in CBR. The maximum positive ΔD (over design, due to a positive variation in EAL) is seen to be about 1.2 inches while the maximum negative ΔD (under design due to negative variation in EAL) is about 2.2 inches.

At this stage it should be recognized that in any construction problem considerable variation in constructed thickness is inevitable. Results of studies made in the



(a) Effect of EAL variations on required thickness



(b) Effect of EAL variations on percent change in required thickness

Notes: $\Delta D = D_i - D_o$

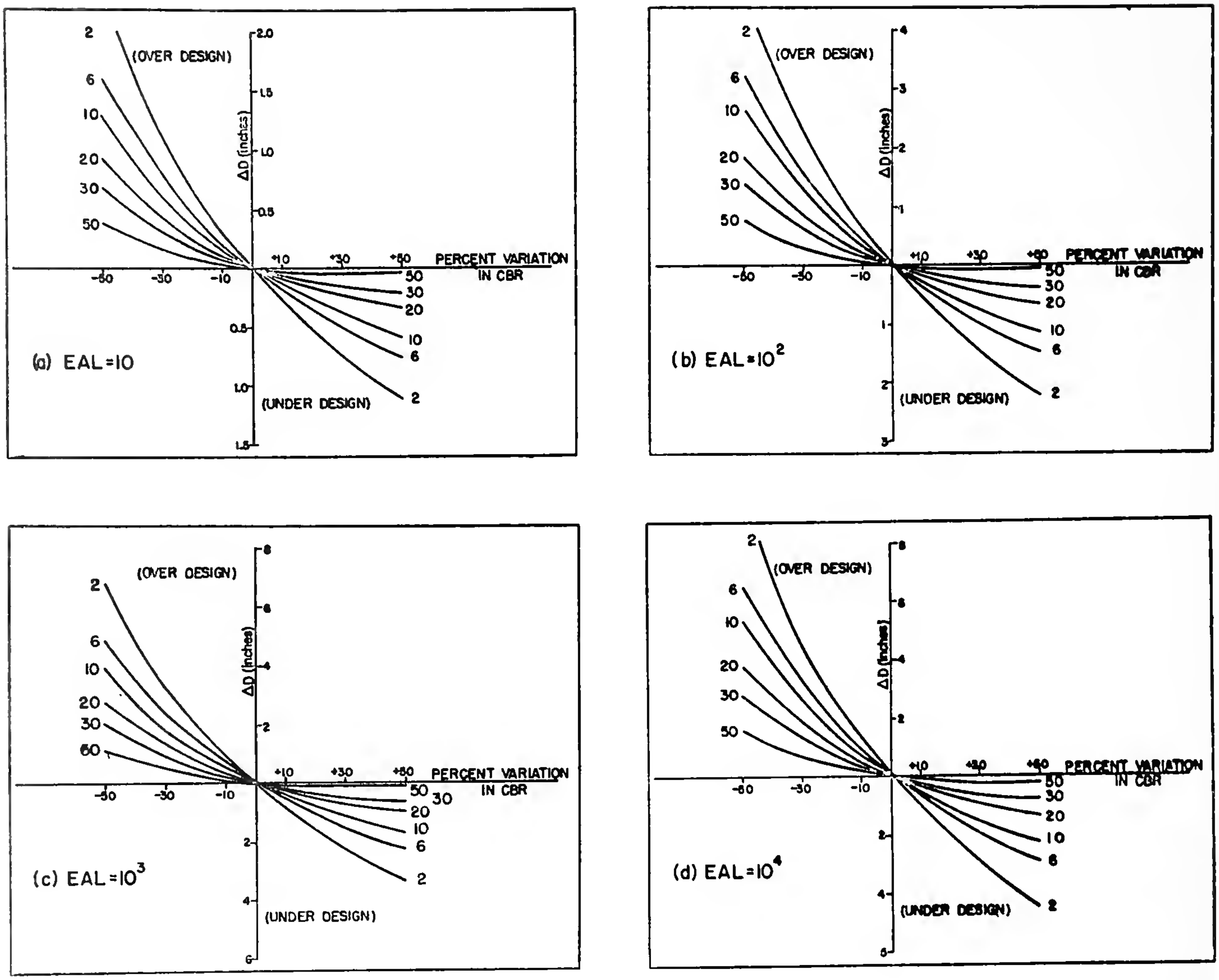
ΔD = Change in required thickness (inches)

D_i = Required thickness for the estimated EAL

D_o = Required thickness for the true EAL

Percent change in $D = \frac{\Delta D}{D_o}$ %

FIG. 3 EFFECT OF EAL VARIATIONS ON PAVEMENT THICKNESS



Note:
$\Delta D = D_1 - D_0$
ΔD = Change in required thickness (inches)
D_1 = Required thickness for the estimated CBR
D_0 = Required thickness for the true CBR
Numbers on curves are CBR values in percent

FIG. 4 EFFECT OF CBR VARIATIONS ON PAVEMENT THICKNESS

United States suggest that using ordinary construction equipment, it is nearly impossible to construct a subbase course or a base course to a tolerance of less than $\pm$ 1.5 inches. This, of course, depends upon many factors including whether electronic devices are used for control of grade. For low volume roads it is recognized that the variations in thickness as given in previous paragraphs (1.2 inches and 2.2 inches) are, for the general case, well within the tolerances that can actually be constructed using ordinary construction equipment.

If one is to consider the percent change in thickness due to variation in EAL as demonstrated in Figure 3(b) it is to be seen that it is dependent upon the magnitude of EAL and again independent of CBR values. For a given percent variation in EAL the percent change in the required thickness increases with a decrease in the magnitude of EAL. Furthermore, it is to be seen that the <u>percent</u> change in thickness varies from a negative value of 40 percent to a positive value of 20 percent. Use of "percent change" in the sensitivity analysis is considered to be a distortion of the true facts since, for low volume roads a 20 percent variation in thickness represents a minimal amount of paving material. For the remainder of the analysis, use is made of the finite value of change rather than percent change.

Effect of CBR Variations. The resulting change in thickness, ΔD, as a function of CBR variations is shown in Figure 4. The change in thickness due to variations in CBR values is dependent upon both CBR and magnitude of EAL. For a given percent variation in CBR, the change in required thickness increases with a decrease in the CBR value. At the same time, ΔD increases with an increase in the magnitude of EAL. Likewise a positive variation in CBR produces less change in the required thickness than negative variations.

The variations in CBR values are seen to have a higher and more significant effect when soil with low CBR's (for example 2% to 5%) are considered than when soils with higher CBR values (20% to 50%) are considered. Hence an important conclusion to be drawn from the above is that the sensitivity of the thickness requirements to variations in CBR and traffic is a function of CBR itself. CBR is known to be a function of material type as well as degree of saturation.

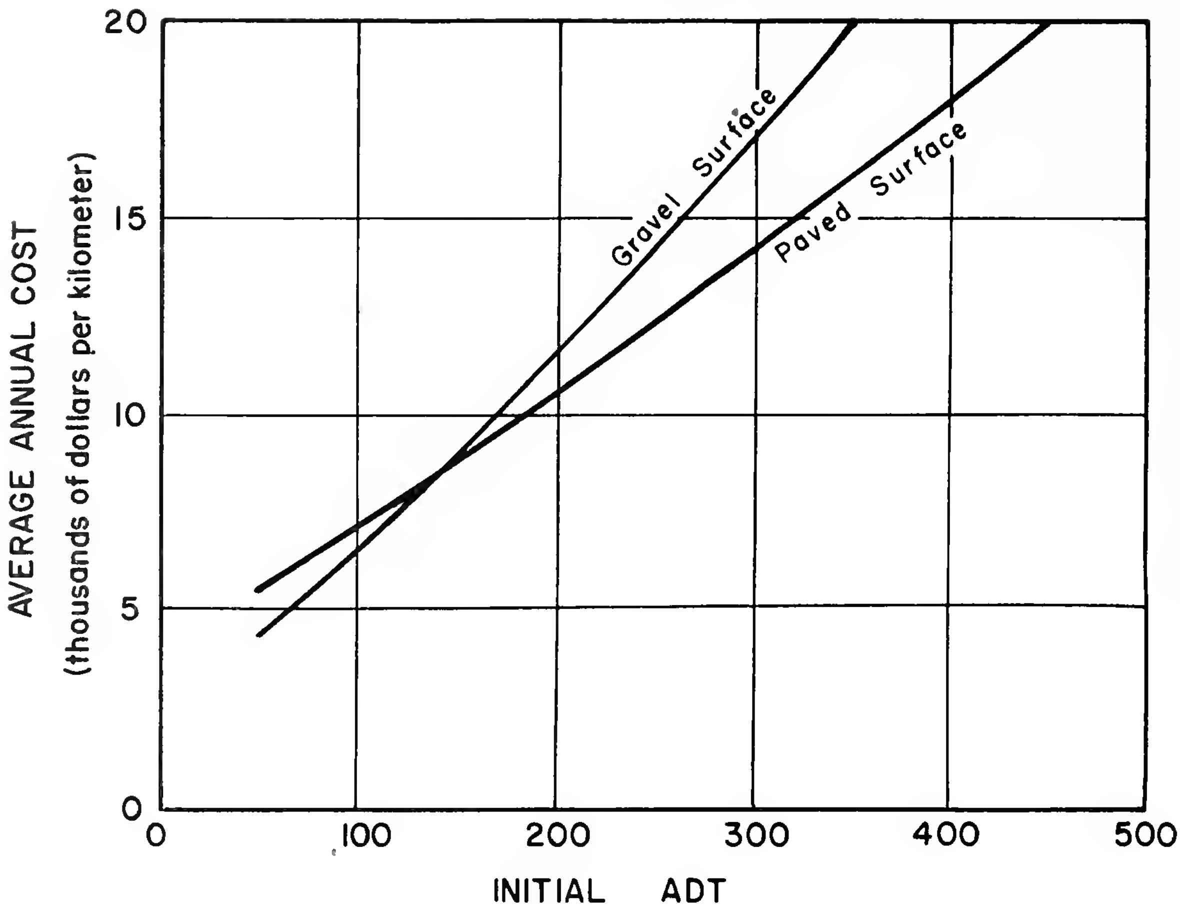
Therefore, as a general statement, in arid climates and for sandy soils variations in CBR are insignificant in the overall analysis. This fact suggests that an estimate of the CBR is all that is required for most design problems. Likewise, the data suggest that approximations of the equivalent axle load (EAL) are all that is required and that there is little need to make a detailed traffic analysis for good subgrade conditions.

Summary, Sensitivity to CBR and EAL. In general, the variation in estimating EAL has relatively less effect on the required thickness than the effect of variation in estimating CBR values. Work reported by Howe (16) has suggested that in developing areas, the general degree of accuracy of estimating EAL lies between 50% and 100%. Howe further suggests there is a need for developing techniques for estimating traffic to higher degrees of accuracy than are presently available. The analysis presented in this paper supports another conclusion by Howe that traffic is a critical factor for low strength subgrades (CBR's between 2% to 5%) but for higher strength subgrades and, incidently for low degrees of saturation as in arid areas, traffic becomes less critical and can, in fact, become insensitive.

Guidelines for estimating CBR values will be presented in subsequent paragraphs.

GRAVEL VS. PAVED SURFACES

Decisions relative to adoption of a paved surface as compared to gravel surfaces depend upon many factors including nuisance from dust, maintenance costs and road user costs. Figure 5 shows an analysis made for a road in Central Africa in which the average annual cost for a gravel surface was compared to the average annual cost for a paved surface. For this particular example, the break-even point (where each surface had equal average annual



Notes:

1. Subgrade CBR = 5%.
2. Traffic growth = 6%.
3. Road user costs of gravel and paved surfaces determined using data from DeWaille.
4. Gravel surface assumed to be 10 cm. thick and replaced every five years.
5. Routine maintenance costs determined using data from Yoder, Ramjerdi, and Grecco.

FIG. 5 EXAMPLE OF COST OF GRAVEL AND PAVED SURFACES

cost) was approximately 140 vehicles per day. This is but one example; the decision for a given locality must be made after a careful analysis of maintenance and road user costs for the conditions at hand. Road user costs are most important. Maintenance costs on gravel surfaces depend upon climate of the area as much as on any other single factor. In areas of high rainfall and where the road bed is subjected to capillary action the gravel surface may deteriorate rapidly while in relatively arid climates the gravel surface may last a longer period of time. On the other hand, the surface may be readily lost in either case due to dusting under the action of traffic.

Oglesby and Altenhofen (22) have suggested that the break-even point for gravel as opposed to paved surfaces is in the vicinity of 100 vehicles per day although they recognize that this is dependent upon many factors and that it varies considerably from locale to locale.

SOIL MAPPING, DESIGN UNITS, SOIL STRENGTH AND SOIL VARIABILITY

It is a basic premise of this paper that low volume roads can be designed on an areal basis. This involves delineation of design units. The design units in turn are determined by soil type, environment and traffic that will use a particular section of road. The next sections of this paper will deal with the matter of selection of design units and how these units can be used for a specific

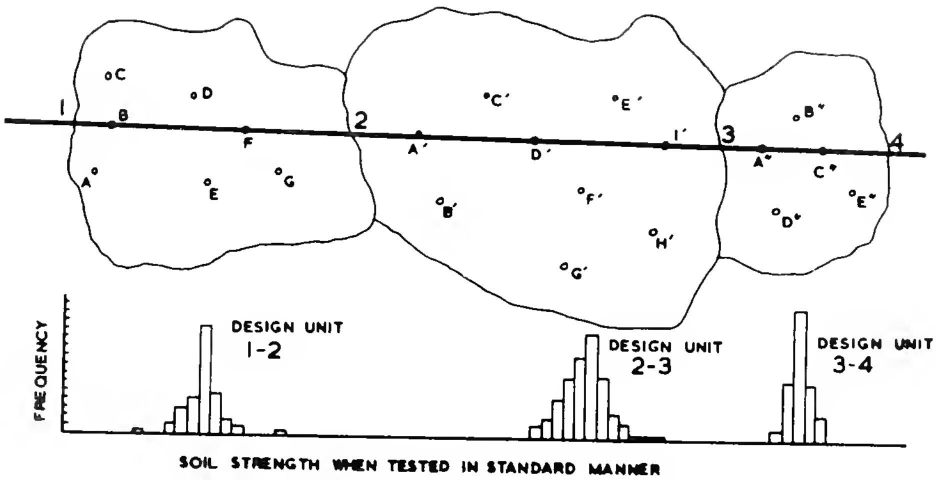
design problem. Summaries will be presented which set forth principles of design which can be adopted for a variety of situations.

The Character of Natural Soil Deposits, Design Units. It has been demonstrated that selection of design units should logically depend upon geology, climate and traffic of the area under consideration. Reference will first be made to the matter of establishing a design unit based upon soils of the area.

Figure 6 shows a generalized representation of pavement design units. These units are generally delineated prior to sampling although in some cases they can be established during the sampling program. They are delineated on the basis of geology, pedology and environment at the site including drainage conditions. In addition, it is necessary to take into account variation of traffic on the road.

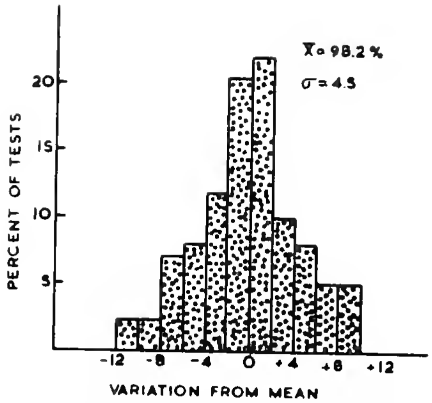
Variability in soil test data will result in any design unit as illustrated in Figure 6(a). Variability of soil test values is dependent upon many factors including the inherent characteristics of soil in place, methods of sampling, method of test and other factors.

The factor of natural soil variance is further complicated by compaction variation as demonstrated in Figure 6(b). This is further compounded by the variation in moisture content as demonstrated in Figure 6(d).



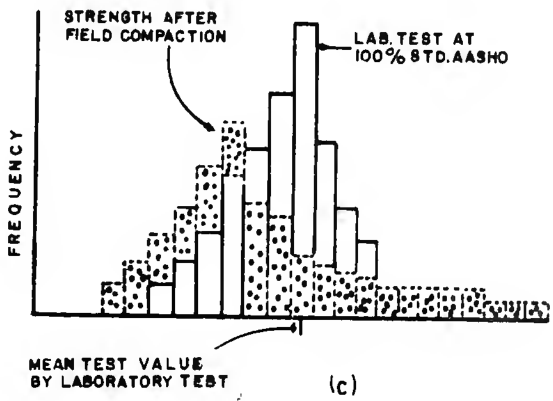
VARIATION OF NATURAL SOIL

(a)

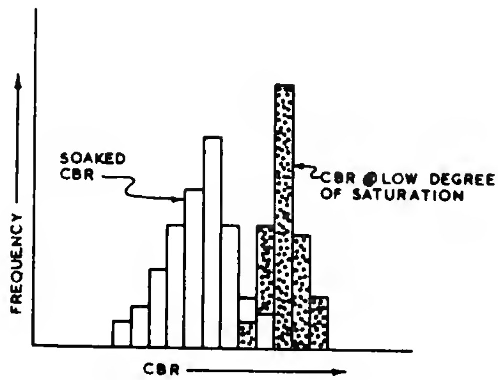


VARIATION OF PERCENT COMPACTION FOR A SUBGRADE

(b)



(c)



EFFECT OF MOISTURE ON DISTRIBUTION OF CBR

(d)

FIG. 6 GENERALIZED VARIABILITY OF PAVEMENT DESIGN UNITS

In summary, delineation of a soil unit requires a comprehensive evaluation of the area under consideration. Full reliance should be placed upon past experiences and test data obtained for the area. It should be recognized that variance of test data will result in any situation. This variance, often times presents a formidable problem to the engineer. However, recognition of this soil variance (both natural and that brought about by construction techniques) is an important phase in the design.

Variation of Test Data. Table 2 shows a summary of basic test data collected by the senior author from a large number of soil types throughout the world. Both average test results and the variation in test results are shown in the table. Of particular consequence is the information shown in the extreme right hand column where the coefficient of variation in CBR is demonstrated. It is seen that the standard deviation averaged about 3 to 5 and, therefore, the coefficient of variation which is utilized in the analysis of test data is dependent upon the mean value. The coefficient of variation is seen to range as high as 60% to 70% and an average coefficient of variation of about 50% can be expected. It is further noted that the coefficient of variation is dependent upon the type of soil under consideration. For sands the coefficient of variation is relatively low and for plastic clays significantly higher values are indicated.

Table 2. Summary of basic test data

Major Division	Area No.	Soil Type	Liquid Limit Mean (%)	Liquid Limit σ	Liquid Limit C.V. (%)	Plast. Index Mean (%)	Plast. Index σ	Plast. Index C.V. (%)	Pass No. 200 Sieve Mean (%)	Pass No. 200 Sieve σ	Pass No. 200 Sieve C.V. (%)	Soaked CBR Mean (%)	Soaked CBR σ	Soaked CBR C.V. (%)
TRANSPORTED	1	Moraine	44	17	39	25	12	48	75	15	21	5	3	60
	2	Alluvium	49	7	14	24	6	24	52	19	36	-	-	--
	3	Granular terrace	47	7	15	22	7	32	48	17	36	6	3	50
	4	Drift (Young)	30	6	20	16	4	25	68	10	15	4	3	75
	5	Moraine	34	11	32	16	9	56	68	23	34	6	3	50
	6	Alluvium	42	9	21	16	8	50	36	13	36	8	3	37
	7	Alluvium	37	7	19	16	5	31	15	5	33	-	-	--
	8	Alluvium (sandy)	39	8	20	16	5	31	34	16	47	7	3	43
	9	Moraine	32	6	19	14	5	36	65	12	18	6	3	50
	10	Drift (Young)	29	10	34	13	7	54	56	16	29	6	3	50
	11	Drift (Old)	30	8	27	13	6	46	57	16	28	5	3	60
	12	Drift (Young)	29	10	34	12	7	58	53	19	36	5	2	40
	13	Drift (Old)	27	5	19	10	5	50	63	15	24	6	3	50
	14	Drift (Young)	20	-	--	9	4	45	67	9	13	7	4	57
	15	Moraine	25	10	40	9	7	78	62	14	22	5	1	20
	16	Alluvium (sandy)	25	6	24	6	5	83	21	10	48	-	-	--
	17	Coastal Plain sand-gravel	22	7	32	-	-	---	34	8	23	-	-	--
	18	Lacustrine sand	N.L.	-	--	N.P.	-	--	8	4	50	23	1	4
	19	Lacustrine sand	N.L.	-	--	N.P.	-	--	7	3	43	19	1	5
	20	Glacial Outwash	N.L.	-	--	N.P.	-	--	26	18	69	16	8	50
	21	Terrace Gravel	N.L.	-	--	N.P.	-	--	14	5	35	55	11	20
	22	Terrace Overburden	N.L.	-	--	N.P.	-	--	66	5	8	--	-	--
RESIDUAL	23	Volcanic	56	9	16	24	7	29	43	19	44	6	3	50
	24	Limestone	46	17	37	23	14	61	95	8	8	8	3	38
	25	Shale	--	--	--	21	6	29	--	-	-	-	-	--
	26	Limestone	39	15	39	20	14	70	88	6	7	5	2	40
	27	Talus	46	8	17	20	6	30	23	11	48	6	2	33
	28	Shale	44	8	18	19	6	31	41	18	44	6	3	50
	29	Shale	47	12	26	18	5	28	27	13	48	9	4	44
	30	Volcanic	41	11	27	16	8	50	32	19	60	7	4	57
	31	Talus	38	2	5	16	16	100	36	9	25			
	32	Igneous	42	7	17	15	10	66	24	10	42	7	5	71
	33	Metamorphic	34	10	29	13	8	61	38	23	60			
	34	Shale	33	6	18	13	4	31	43	22	51	7	2	29
	35	Granite	--	-	--	11	4	37	25	4	16			
	36	Gneiss	28	8	29	9	6	66	25	16	64	20	9	45
	37	Shale	32	3	9	9	3	33	33	15	45			
	38		23	14	61	8	8	100	27	13	46	--	-	--
	39	Gneiss-Schist	30	4	13	8	3	37	48	13	27	38	18	47
	40		21	7	33	7	7	100	27	13	48	--	-	--
	41	Sandstone	28	5	18	7	4	57	47	13	28	--	-	--
	42		26	2	8	6	4	66	34	8	24	--	-	--
	43		29	2	7	6	4	66	25	14	56	--	-	--
	44		27	9	33	-	-	--	49	14	29	--	-	--

Selection of a Specific Test Value. A study was made of the variance that might be expected within soil deposits and a least cost analysis was made of these data to determine the percentile test value that might be appropriately selected for design (32). Figure 7 shows data which can be used as a guide for selecting test values from an array of tests that have been obtained for a soil area. Data are presented upon the basis of soil type as well as a factor which is termed the "Cost Ratio". The Cost Ratio is defined as the ratio of unit maintenance costs on a spot-to-spot basis as opposed to original main line construction. For example, if an original pavement when paved on a mass production basis costs x dollars, and, in particular if the road is in a remote area, the unit cost for patching this road on a small area basis can be considerably more than the original unit cost. It is suggested that for remote areas the Cost Ratio can be as high as 9 or 10.

Use of Figure 7 is relatively simple and is demonstrated as follows. Assume that the variation in compaction typically might result in a standard deviation of 5 percentage points of compaction, and that it is anticipated the total traffic (EAL) to use the road over the design life will be 5×10^3. Assume further that the subgrade will become saturated during its life and that the cost ratio is 10.

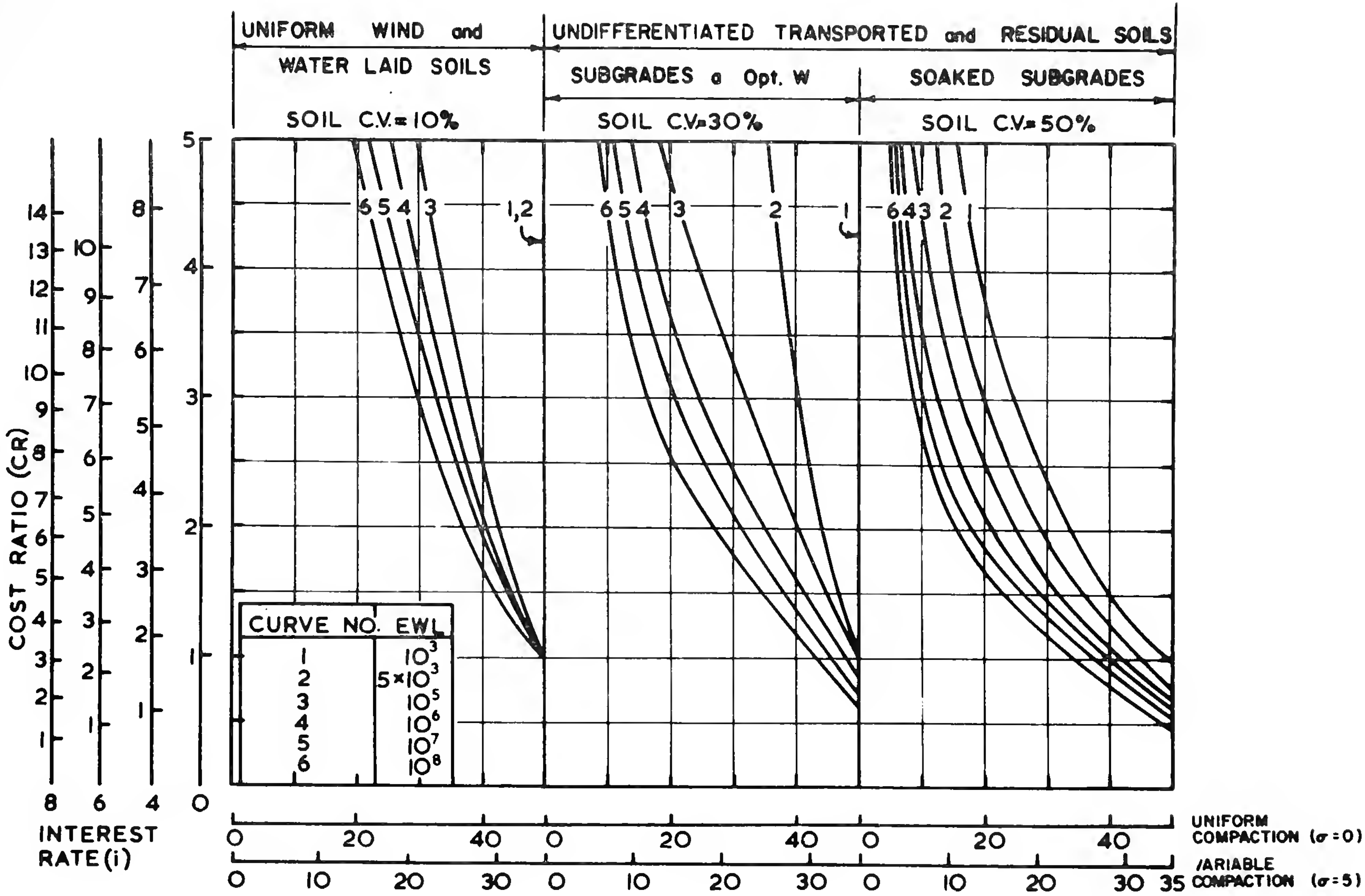


FIG. 7 PERCENTILE TEST VALUES FOR LEAST COST DESIGN

For this example, curve No. 2 on the extreme right-hand side would be used. Entering the vertical scale at a Cost Ratio of 10 and at an assumed interest rate of 8% it is seen that the percentile test value to use for design would be about 12%. This means that 88% of the test values would be lower than the design value and that some distress to 12% of the road can be expected. This distress to certain parts of the road is an inevitable problem and must be coped with by adequate maintenance programs. This factor is taken into account in setting up the guidelines shown in Figure 7.

For uniform wind and water laid soils and for relatively low amounts of traffic it can be seen that the optimum design value is an <u>average</u> strength value for the area as demonstrated on the left-hand side of Figure 7.

<u>Effect of Degree of Saturation</u>. Historically, engineers have performed the laboratory California Bearing Ratio test after soaking the sample for four days. The soaked CBR test is intended to represent the worst condition that might exist under the pavement. For arid and semi-arid climates this might be too restrictive and it is suggested that the test should be made on samples which are compacted at optimum moisture content or at some value less than this.

In the study mentioned above the degree of saturation was evaluated and these data are illustrated in Figure 8. It has been demonstrated (32) that soils that exist at optimum moisture content or lesser degrees of saturation during the life of the pavement have lower coefficients of variation and, hence, the percentile test value to use in design approaches the average value. This is demonstrated in Figure 8 where it is seen that the coefficient of variation for soils having CBR values up to 20% when tested in a soaked condition approach a coefficient of variation of up to 70%, but when tested at 80% degree saturation this figure drops down to about 45 percent. Hence, it must be concluded that in arid and semi-arid areas the average test value is the one to be used rather than some intermediate value.

Guidelines for Soil Exploration and Testing. Table 3 shows suggested guidelines for soil exploration and selection of design value for low and medium volume roads. The table is broken down on the basis of type of soil. The factor of traffic, number of samples required to adequately define a soil unit as well as percentile test value to use for design are presented in the table.

It is suggested that the guidelines in Table 3 can be used to fit a wide variety of conditions. As a general rule, for soaked conditions which include clays under areas of high rainfall, the 20th to 35th percentile test value may be used but for gravels and sands in arid climates,

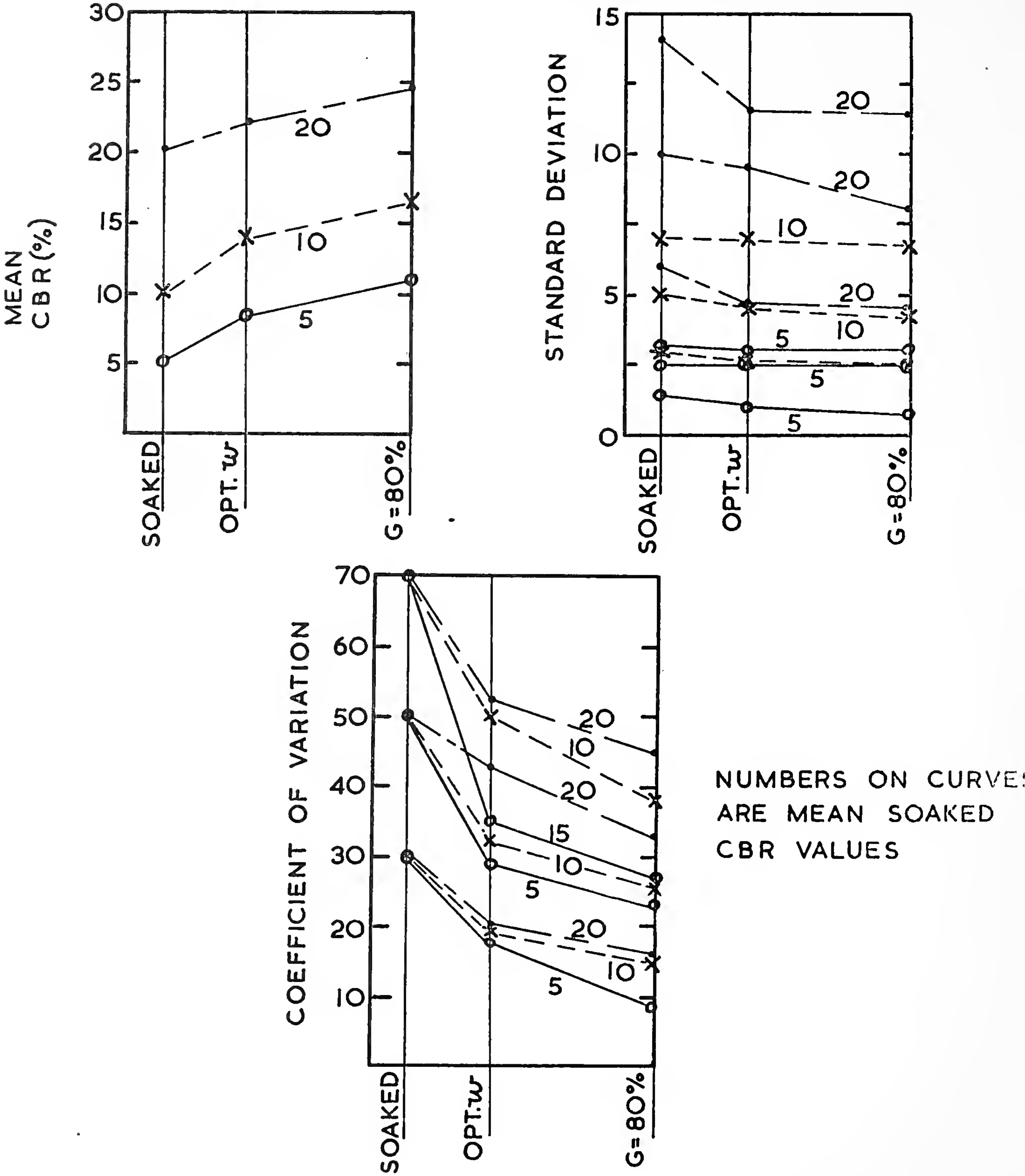


FIG. 8 EFFECT OF DEGREE OF SATURATION ON CBR AND CBR VARIATION FOR SEVERAL TYPICAL SOILS

Table 3 . Suggested Guide Lines for Soil Exploration and Selection of Design Value for Low and Medium Volume Roads

Item	Undifferentiated Transported Soils (tills, clays etc.)	Gravels and Sands	Wind and Water Laid Soils*	Residual Soils
Low traffic (25-100 ADT)				
Basis soaked values				
N*	10 - 15	5 - 10	3 - 5	10 - 15
% tile (low C.R.)**	30	35	35	30
% tile (high C.R.)	20	30	30	20
Basis opt. w				
N	5 - 10	3 - 7	3 min.	5 - 10
% tile (low C.R.)	35	40	35 - 50	35
% tile (high C.R.)	30	35	35 - 50	20
Med. traffic (100-1000 ADT)				
Basis soaked values				
N	15 - 25	10 - 15	3 - 5	15 - 25
% tile (low C.R.)	30	35	35	30
% tile (high C.R.)	15	30	30	15
Basis opt. w				
N	7 - 12	5 - 10	3 min.	7 - 12
% tile (low C.R.)	35	40	35 - 50	35
% tile (high C.R.)	20	35	35 - 50	20

* Number samples required
**Percentile test value used for design

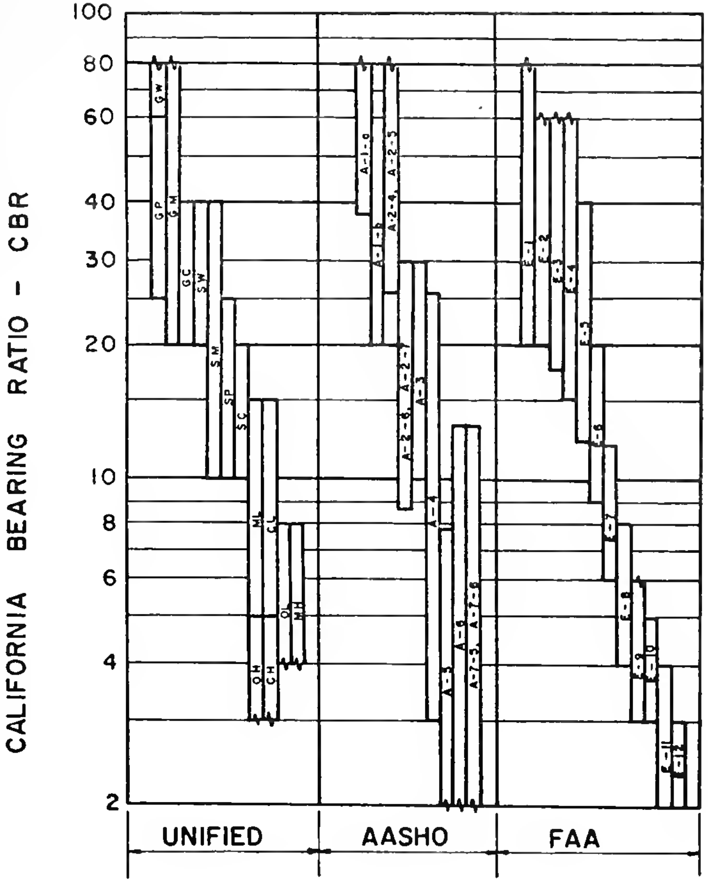
the average test values will adequately define the soil area under consideration.

With regard to the number of samples required to define a soil area this is seen to also vary with the type of material. For arid and semi-arid areas the number of samples decreases to a relatively few number of samples whereas for soaked tests the number can be as high as 25 samples. The primary point made is that in sandy and arid areas a relatively few number of samples will adequately define the area.

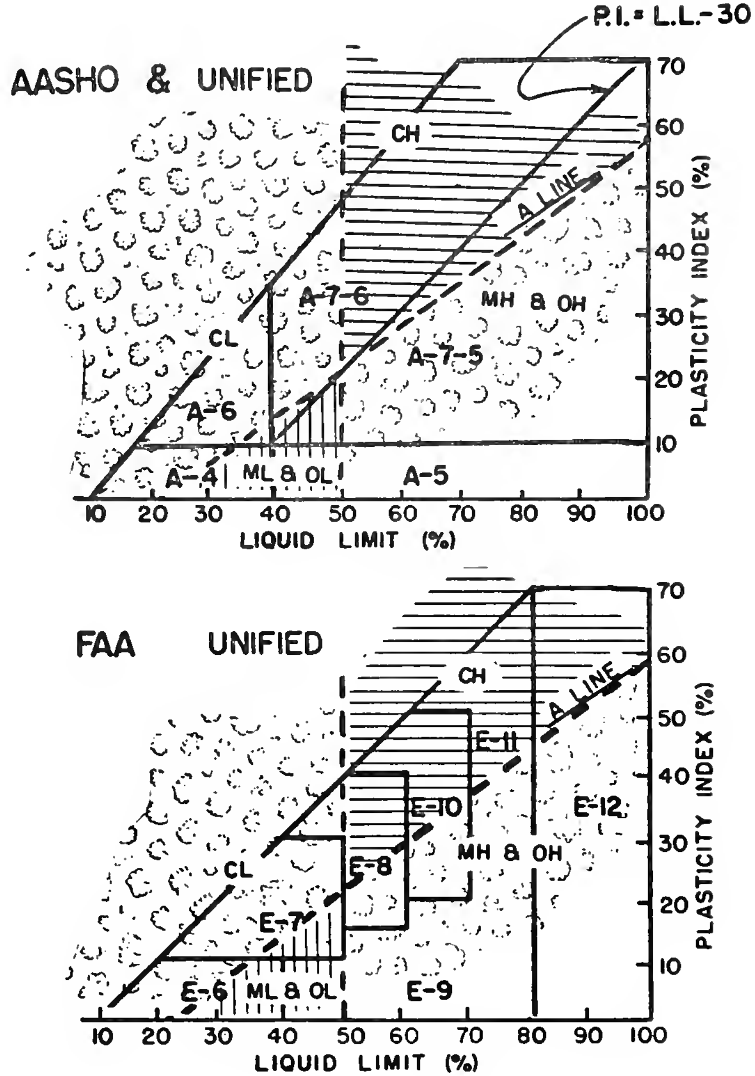
USE OF SOIL CLASSIFICATION AS AN AID IN DESIGN

Obviously for many routine jobs the cost of performing a large number of strength tests is often prohibitive. This is particularly true of low volume roads wherein minimum thicknesses might be used for a given amount of traffic. For these situations soil classification offers a tool that the engineer can use in estimating soil behavior.

Figure 9 shows interrelationships among several classification systems commonly in use throughout the world. In the left-hand portion of the figure (b) a comparison is made of three classification systems, i.e., the AASHO system, the FAA system and the Unified system. It is seen that the AASHO and Unified systems rely heavily upon a line which separates the high and low plasticity materials.



a) INTER-RELATIONSHIPS, CBR, AND THREE SOIL CLASSIFICATION SYSTEMS. (MODIFIED AFTER PCA AND OTHERS.)



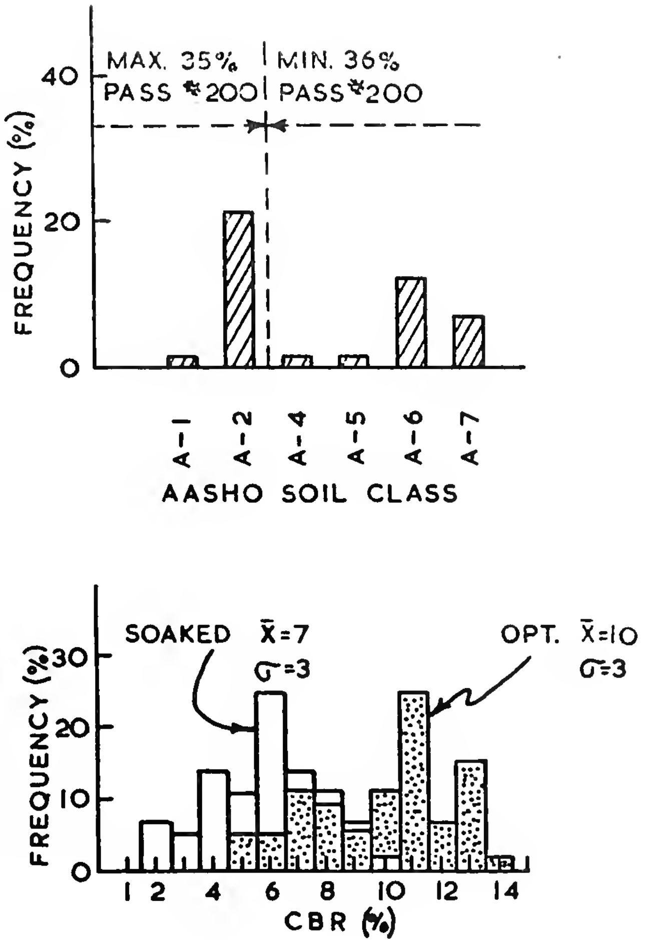
(b) OVER-LAY COMPARISONS

FIG. 9 AASHO, FAA AND UNIFIED SOIL CLASSIFICATION

Figure 9(a) shows correlations that have been established among the classification systems and California Bearing Ratio. This figure can be used with some reliability to estimate the California Bearing Ratio if the soil classification is known.

Fairly reliable estimates of the California Bearing Ratio can be made if the soil areas are delineated and if sufficient samples are obtained to classify the soil adequately.

<u>Example of Using Soil Classification</u>. Figure 10 demonstrates a technique which can be used for estimating the thickness of a low volume road by first estimating the CBR and selecting a percentile CBR value as was demonstrated in previous paragraphs. Figure 10(a) shows soil classification data for a particular alluvial soil. The frequency distributions for actual CBR's, both in the soaked and unsoaked conditions, are shown in the lower portion of the graph. Referring to the soil classification data in Figure 10(a) it can be assumed that the A-6 would perhaps be the percentile test value to use for the soil. In Figure 10(b) a comparison of design thicknesses for two volumes of traffic (25 ADT and 1000 ADT) using actual CBR values as opposed to the correlations as estimated from the classification data are shown. For the soaked condition some error is involved in estimating the CBR but for tests made at optimum moisture content (corresponding to fairly dry areas) and for low volume roads, identical answers are arrived at using classification data or tested CBR.



DISTRIBUTION OF SOIL CLASSIFICATION AND CBR FOR AN ALLUVIAL SOIL

(a)

(b) Use of Soil Classification as an Aid in Selecting Design Values

Design Values		Traffic			
		25 ADT		1000 ADT	
Basis	Design CBR	t	error	t	error
40 soaked samples	5%	11"	--	21"	--
20 soaked samples	5%	11"	0	21"	0
10 soaked samples	5%	11"	0	21"	0
% tile PI	7%	9"	18%	17"	19%
A-6 soil class (5 samples)	6%	10"	9%	18"	14%
40 opt. w samples	10%	7"	--	13"	--
20 opt. w samples	10%	7"	0	13"	0
10 opt. w samples	10%	7"	0	13"	0
% tile PI	12%	6"	14%	11"	15%
A-6 soil class (5 samples)	11%	6.5" (use 7)	--	12"	8%

FIG. 10 EXAMPLE OF USING SOIL CLASSIFICATION AS AN AID IN SELECTING DESIGN VALUES

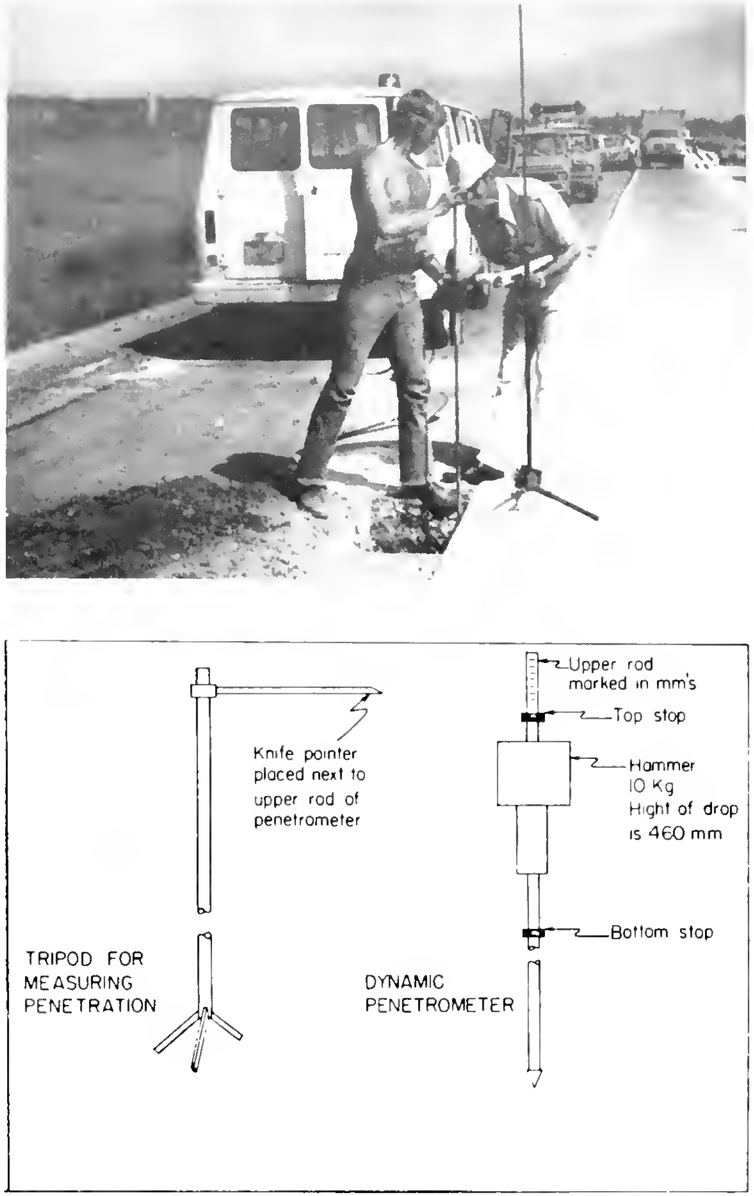
The data for this particular example taken from an actual road in Central America, demonstrate that for low volume roads estimations of the CBR on the basis of classification data are often justified especially in arid and semi-arid regions.

"QUICK" TESTS FOR ESTIMATING CBR

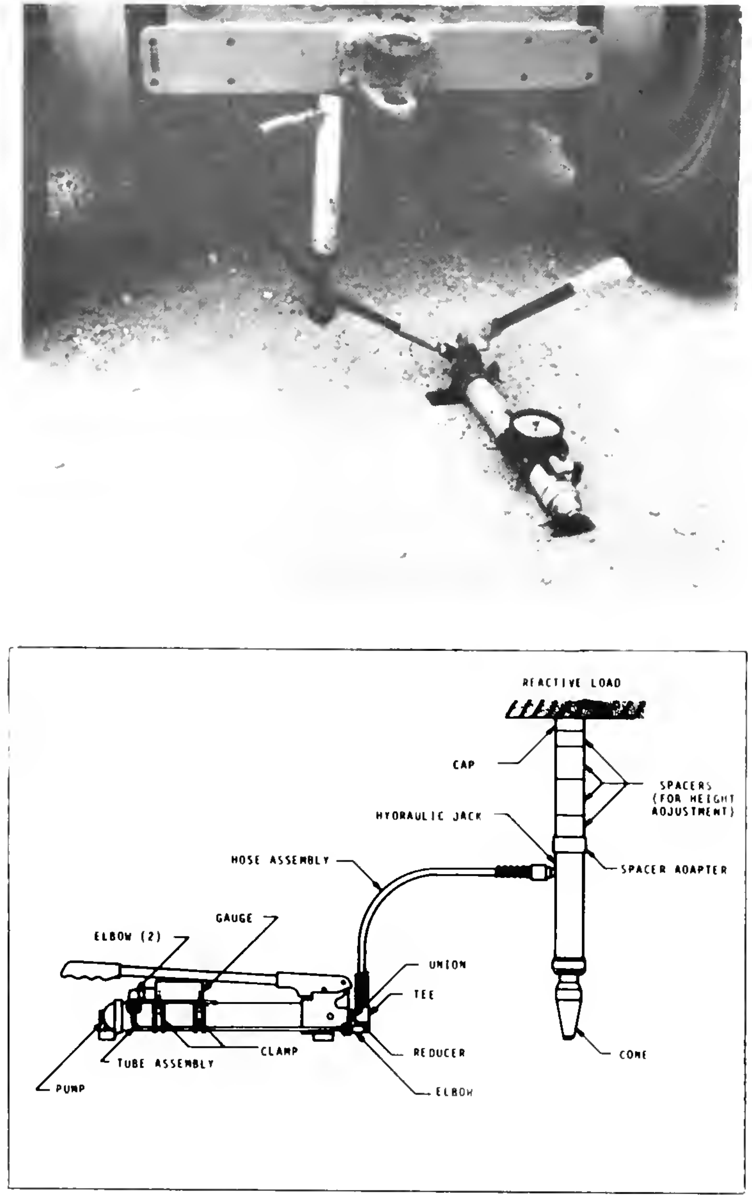
The California Bearing Ratio test is a fairly time consuming test and many times is quite expensive. Figure 11 demonstrates two methods that are reliable for measuring the in-place CBR's of both fine grained and coarse grained soils. Figure 11(a) shows a dynamic cone penetrometer which was developed in South Africa based upon concepts first developed in Australia (30). This dynamic cone penetrometer has proven successful and it greatly shortens the time required for testing and it offers a ready means of estimating CBR of fine grained materials. A large number of tests can be obtained by means of the cone penetrometer.

Figure 11(b) shows a large cone device which is a reliable instrument for estimating CBR of granular materials. This particular device was devised for measuring CBR of existing air fields and it has proven successful in estimating CBR of existing gravel roads.

The two penetration devices shown in Figure 11 are offered as a means of obtaining a large number of measurements in an economical manner from which a design can be



(a) DYNAMIC CONE (FINE GRAINED SOILS)



(b) HIGH LOAD CONE (GRANULAR SOIL)

FIG. 11 TWO METHODS OF RAPID CBR TESTS

obtained. The use of these instruments permits making a large number of tests thereby reducing errors that might be introduced as a result of variability. The percentile test values, then, shown in the guidelines of Table 3 can be used for obtaining design CBR values.

In light of the sensitivity analysis that has been presented relative to the effect of CBR and equivalent axle load, the data suggest that the engineer must consider whether or not he is dealing with the fine grained, fairly weak subgrade or whether he is dealing with a high CBR material. For the later materials, the quick test shown in Figure 11 certainly should suffice and full use of soil classification as a method of estimating design values should be made. It is only when poor subgrades are encountered that detailed studies are justified.

TECHNIQUES FOR ESTIMATING TRAFFIC

It will not be the intent here to discuss methods of traffic counts, techniques for weighing axles, etc., but it appears pertinent to mention simplified techniques for evaluating the effect of traffic.

Extensive use has been made throughout the world of the equivalency factors (damage factors) as proposed from the AASHO Road Test (1). A design problem is presented at the end of this paper wherein use is made of these equivalency factors. It is suggested that if there is any doubt at all, use should be made of equivalency factors such as proposed by the AASHO Interim Guide but that these can be simplified for any given area.

Methods of calculating the accumulated axle loads over the life of the pavement are shown in equations 12 and 13. Equation 12 is a general form for accumulated axle loads if the initial EAL_0 is known. It is further suggested, however, that general use can be made of Equation 13 and that constants can be derived for a given area by statistically sampling roads within the area and determining typical EAL values that can be expressed as a function of the average daily traffic. This procedure has been adopted by many states in the United States for estimating EAL for various classes of roads. The technique permits the engineer to classify the roads on the basis of one, two, or perhaps three categories and from these, estimates of the EAL can be made using traffic counts on the road.

SUMMARY DISCUSSION

Previous paragraphs of this paper have presented several basic concepts relative to the design of low volume roads. It is extremely difficult to set down hard and fast rules that might apply to a variety of conditions. The engineer must analyze all of the problems for his specific area and from these make a judgement on the design which is to be adopted.

Nevertheless, there are guidelines that can be used to assist the engineer in making the decisions necessary to finalize the design. The following paragraphs summarize

some of the important factors which were discussed in the paper and in some cases guidelines are provided with the hope that these will assist the engineer in practical problems that might be encountered.

<u>Method of Design</u>. The reader is referred to Figure 1 which summarizes in schematic form the steps that are followed in designing a road. The input variables include load and traffic analysis, environmental factors (moisture content, etc.) and evaluation of materials. Variability of these factors are an inevitable part of the process and this variability must be taken into account at some stage in the design. In the second step, the engineer decides upon design values and from this selects a structure to handle the traffic under consideration. As a last step, through a checking process, he checks his design against the original assumptions taking into account costs including initial and maintenance costs. Regarding the input variables outlined in Figure 1 this can best be accomplished on an areal basis wherein design units are established. These design units are based upon type of soil, environment of the area and traffic that will use the facility.

An estimate can be made of the probable CBR of the design units by use of classification data such as shown in Figure 9. It is also necessary to make an estimate of the probable degree of saturation that will exist in the subgrade. Tecbniques for accomplishing this have not been discussed in this paper. It is suggested that techniques

proposed by the Road Research Laboratory in England can be used for this phase.

After an estimate of the probable CBR is made, data in Table 2 can be used to estimate the coefficient of variation. Figure 7 of the paper presents guidelines for selecting a design CBR. If exact data are deficient in the design problem the following can be used as guides

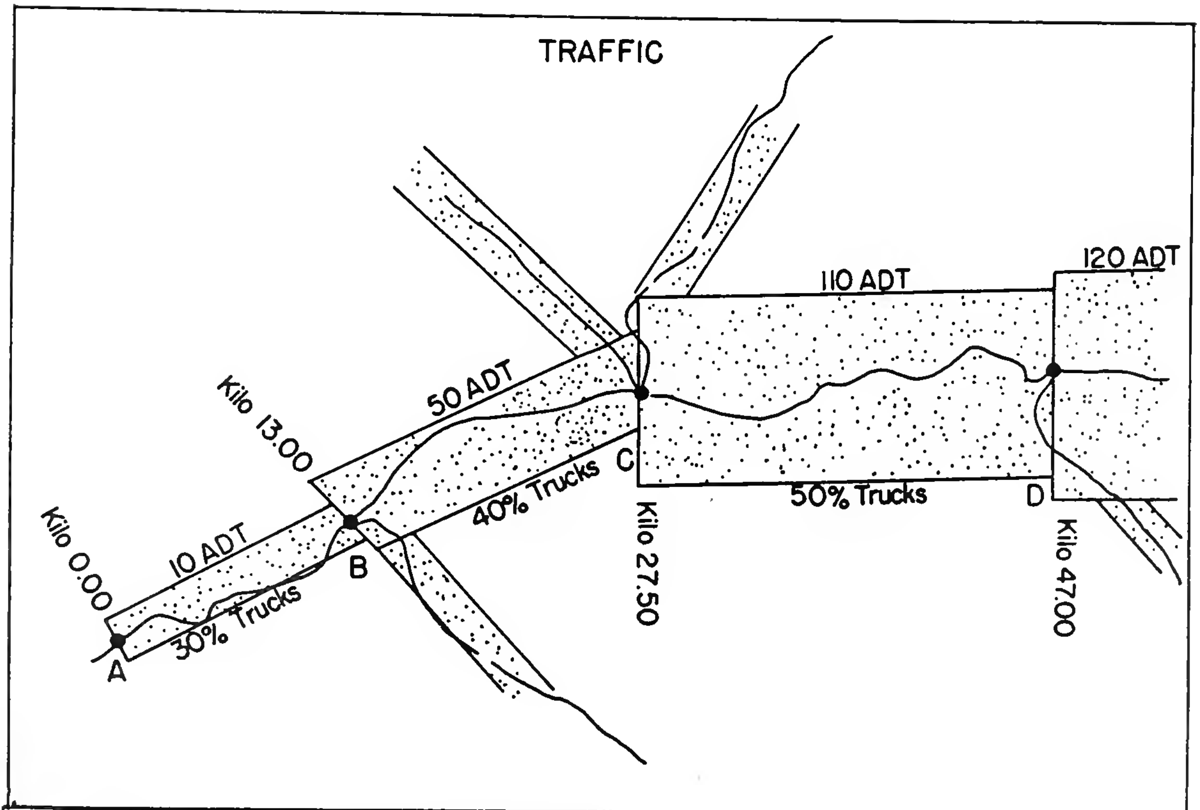
1. Poor subgrades (CBR 2-5%). Make a detailed survey using as a minimum the quick methods of estimating CBR shown in Figure 11 or preferably using laboratory tests. The lower 75th to 80th percentile value may be used for design.
2. Fair subgrades (CBR 5-15%). For these materials the quick methods of estimating CBR's can be used with some reliability; classification data can also be used.
3. Good subgrades (CBR above 15%). Classification data with an occasional check on a strength is sufficient for these cases. The average design value can be used.
4. Arid climates. Average test results may be used for design.
5. Areas of high rainfall (particularly in clayey soils). For this case percentile test values as given in Figure 7 should be used for design.

The next step in the design is to estimate traffic. Use can be made of Equations 12 and 13. Use of equivalency factors as proposed by AASHO is recommended. However, standard factors can be developed for any area and these applied to Equation 13.

For low volume roads stage construction is doubtless the most economical to adopt for most cases. Data in Table 1(b) suggest that for low interest rates, it may be more economical to go for longer periods of design but as a general rule stage construction (less than 10 years) appears to be the most economical approach. If in doubt an economic analysis can be made using standard techniques.

An estimate of the average daily traffic must be made. For ADT values less than about 50 vpd, unpaved surfaces can be assumed to be the most economical, ADT values between 50-100 vpd should be analyzed on an economic basis to determine whether a surface is warranted and for ADT values above 100 vpd paved surfaces of one type or another are generally the most economical.

Design Alternates. To illustrate the above an example of a design problem is presented in subsequent paragraphs. This road passes over residual soils as shown in Figure 12 although transported sands and gravels are noted in several locations. It is to be seen that the traffic increases progressing from left to right on the figure and that the design units as suggested in above paragraphs are based upon an analysis of the soil as well as the traffic at the location.



GENERAL PROFILE
AND
BASIC SOILS

Section No.	Stations (Kilometers)	Soil Type	Design CBR
1	0.00- 6.00	Sandstone	10%
2	6.00- 12.00	Sandstone-shale	10%
3a	12.00- 13.00	Gravel	25%
3b	13.00- 15.95	Sand	15%
4a	15.95-27.50	Clay	4%
4b	27.50-36.00	Clay	4%
5	36.00-40.00	Sandstone-shale	10%
6	40.00-46.25	Sandstone	11%

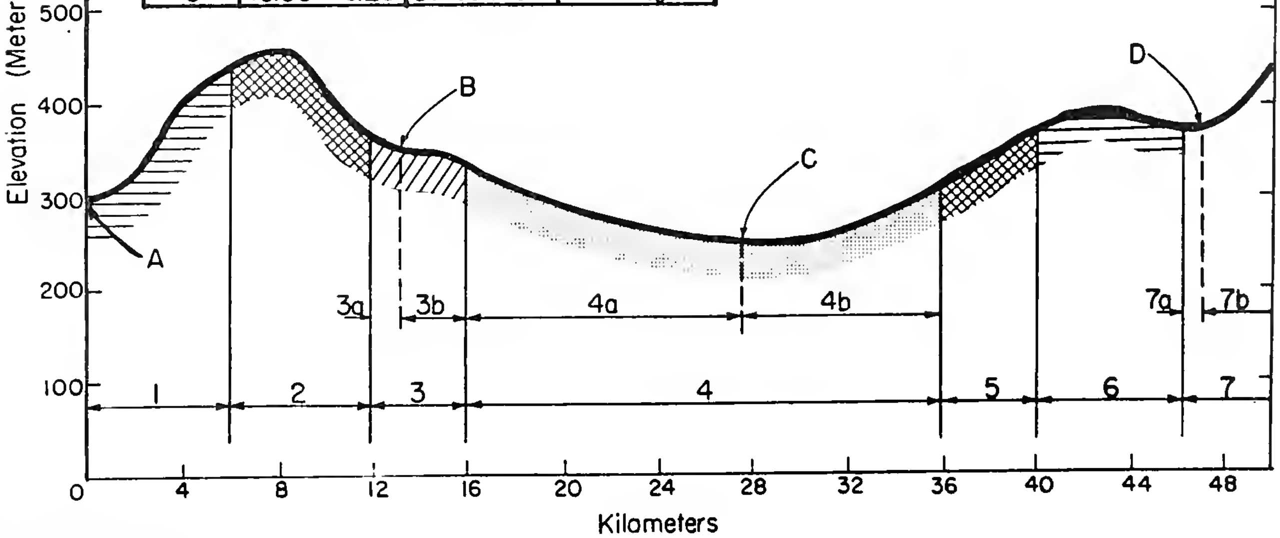


FIG 12

Rainfall on the left hand side of the map up to about kilometer 8 is relatively low and increases beyond this point. The soils of section No. 4 are clay. Field surveys have demonstrated that because of a relatively high water table along with a relatively high rainfall, the soaked CBR should govern the design.

Figure 13 shows a detailed analysis of Section 4 along with an estimate of the EAL that will use the road at this particular site. The computations shown in the right hand column utilize Equations 12 and 13 and it is assumed that the traffic weights as given in the table (average of four different weighings) govern the design.

Referring to the lower part of the figure, it is seen that the existing road, which is a gravel surface, has shown relatively poor performance and there are locations where water is seeping from the road during certain periods of the year. As a part of the design it will be necessary to take care of the subgrade and to account for these soft spots that develop due to the wet subgrade condition.

Therefore it must be recognized that design not only takes into account the thickness of the pavement structure but extreme care must be exercised in preparing the subgrade as well as selection of materials which will be put into the pavement structure. For purposes of this discussion, and to conserve space, this will not be discussed in detail.

Equivolent axle load computation (p = 2.0 and SN = 4)

Axle Load (kips)	Single Axles per 100 Trucks			Tandem Axles per 100 Trucks		
	Number	F	NxF	Number	F	NxF
Under 3000	75.3	0.0002	0.02			
3 - 5	29.9	0.002	0.06			
5 - 7	10.5	0.01	0.11			
7 - 9	3.4	0.03	0.10			
9 - 11	4.2	0.08	0.34			
11 - 13	3.0	0.18	0.54			
13 - 15	4.1	0.35	1.43	0.1	0.03	0.01
15 - 17	9.3	0.61	5.78	0.5	0.05	0.03
17 - 19	11.0	1.00	11.00	1.5	0.08	0.12
19 - 21	8.0	1.55	12.40	2.0	0.12	0.24
21 - 23	5.0	2.31	11.55	3.6	0.17	0.61
23 - 25	1.1	3.33	3.66	4.2	0.25	0.11
25 - 27				8.4	0.35	2.94
27 - 29				9.2	0.48	4.41
29 - 31				5.0	0.64	3.20
31 - 33				1.2	0.84	1.00
33 - 35				0.8	1.08	0.86
35 - 37				0.4	1.38	0.65
37 - 39				0.2	1.72	0.34
39 - 41				0.1	2.13	0.21
41 - 43				0.1	2.62	0.26
Totals			46.99			14.99
Total equivalent 18,000 pound single axles per 100 trucks on road 46.99 + 14.99 = 61.98						

Classification of traffic on the road	
Passenger cars	= 50%
Trucks	= 50%
Two axle	= 40%
Semi-trailers	= 10%

For 5.5% traffic growth

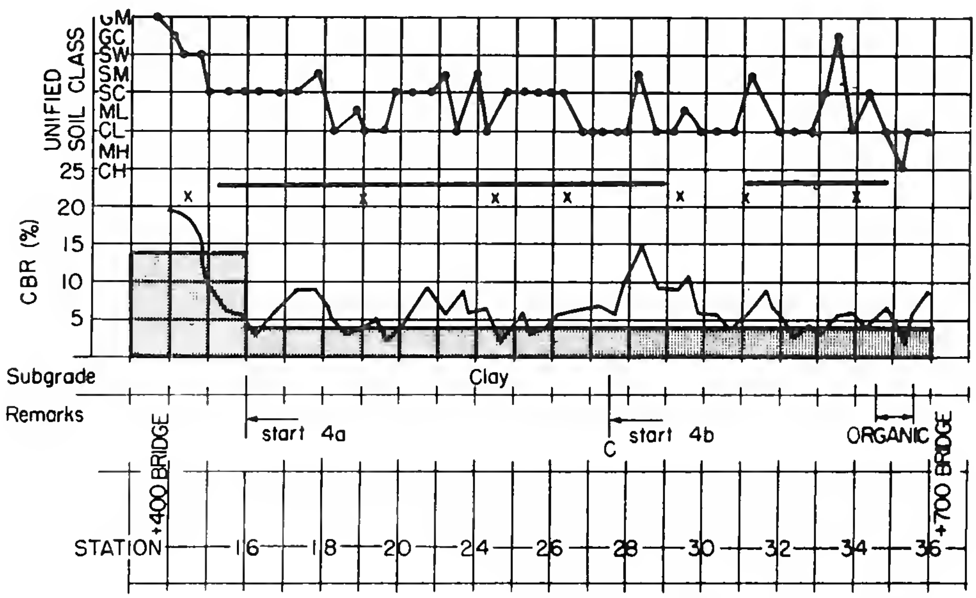
$$\Sigma EAL = \frac{EAL_0(365)}{\log_e(1+i)}\left[(1+i)^n - 1\right]$$

$$\Sigma EAL_{10} = \frac{EAL_0(365)}{0.0535}\left[(1.055)^{10} - 1\right] = EAL_0\,(4{,}830)$$

$$\Sigma EAL_{20} = \frac{EAL_0(365)}{0.0535}\left[(1.055)^{20} - 1\right] = EAL_0\,(13{,}709)$$

Initial Daily EAL_0 =

$$= \frac{61.98}{100}(0.5)\left(\frac{ADT}{2}\right) = 0.155\,(ADT)$$



NOTES: Heavy line shows locations of poor performance.

x indicates locations of water seepage.

FIG. 13

The design CBR for Section 4 is shown as the shaded portion in the figure and is taken to be 4 percent. Table 4 shows a summary of the design data for the entire road under consideration. Table 4(a) shows the ADT values for 10 and 20 year periods assuming a traffic growth of 5-1/2%. It is to be seen that up to a ten-year life, the ADT on Section 4a will be less than 85 vehicles per day. Hence, a basic design premise would be, that up through Section 4a no surface will be applied but rather a gravel road will be used. This decision, however, would need to be modified on the basis of the soils at the site. The soils in 4a are plastic with low CBR, and it may be well to protect this soil from surface moisture infiltration by using on a surface treatment. For this particular situation however, and in the interest of conserving money the decision was made to use a gravel surface. Beyond Section 4b a surface treatment is to be used. The thickness of surface as shown in Table 4 was determined by means of Equation 14 and is that of the Texas Highway Department (27).

$$ds = 1.606 - 1.781 \log EAL + 0.311 \log^2 EAL \qquad (14)$$

Considering the 20 year design it is seen that at some time, and for the more heavily trafficked sections beyond kilometer 27.50 to the right hand side, a relatively greater amount of asphalt surface will need to be applied to account for fatigue of the surface.

TABLE 4 . DATA FOR EXAMPLE PROBLEM

(a) ADT PER SECTION

Section No.	Stations	Initial ADT	10 Years $(1+i)^n$	10 Years ADT_{10}	20 Years $(1+i)^n$	20 Years ADT_{20}
1	0.00-6.00	10	1.708	17	2.917	29
2	6.00-12.00	10	1.708	17	2.917	29
3a	12.00-13.00	10	1.708	17	2.917	29
3b	13.00-15.95	50	1.708	85	2.917	146
4a	15.95-27.50	50	1.708	85	2.917	146
4b	27.50-36.00	110	1.708	188	2.917	321
5	36.00-40.00	110	1.708	188	2.917	321
6	40.00-46.25	110	1.708	188	2.917	321

(b) SUMMARY OF DESIGN DATA FOR ALL SECTIONS*

Section No.	Stations	Design CBR	Initial APT	10 Year Design EAL	10 Year Design Surf.	10 Year Design Total Thickness	20 Year Design EAL	20 Year Design Surf.	20 Year Design Total Thickness
	(Kilometers)	(%)			(in.)	(in.)		(in.)	(in.)
1	0.00-6.00	10	10	$7.5x10^3$	-	8	$2.1x10^4$	-	8+
2	6.00-12.00	10	10	$7.5x10^3$	-	8	$2.1x10^4$	-	8+
3a	12.00-13.00	25	10	$7.5x10^3$	-	4	$2.1x10^4$	-	4+
3b	13.00-15.95	15	50	$3.8x10^4$	-	7	$1.1x10^5$	ST	8
4a	15.95-27.50	4	50	$3.8x10^4$	-	17	$1.1x10^5$	ST	19
4b	27.50-36.00	4	110	$8.2x10^4$	ST	18	$2.4x10^5$	1-1/4	21
5	36.00-40.00	10	110	$8.2x10^4$	ST	10	$2.4x10^5$	1-1/4	11
6	40.00-46.25	11	110	$8.2x10^4$	ST	9	$2.4x10^5$	1-1/4	10

*Notes: Total thickness values obtained by Equation 13. The National Crushed Stone Association design method is basically the same. The NCSA curves, however, are based on traffic classification. Use of the equation although approximate offers more flexibility in use. Thickness values are the same in both methods.
The minimum surface requirements as the Texas values suggested by McDowell and are estimated using equation 14.

<u>Summary</u>. This paper has set down the relative effects of some of the factors that must be considered in the design of low volume roads. It has been demonstrated that some of the factors are relatively insensitive and therefore rough estimates of these are sufficient for most cases. It has further been suggested that the sensitivity of these factors are dependent upon a number of factors including rate of interest, subgrade type and EAL. Use of high rates of interest has the effect of giving preference to deferring expenses to a later date and, hence, stage construction.

For poor subgrades the factors become critical and it is suggested that detailed analysis should be made for these sitautions. For higher strength subgrades, however, rough estimates are generally sufficient and the factors become relatively insensitive.

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